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Geomorphic restoration and stabilization of a reach of Stevens Creek

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Department of Civil Engineering

I hereby recommend that the
SENIOR DESIGN PROJECT REPORT prepared
under my supervision by

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TRAVIS GIFFEN
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GEOMORPHIC RESTORATION AND STABILIZATION OF A REACH OF STEVENS CREEK

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BACHELOR OF SCIENCE IN CIVIL ENGINEERING

[signature]	6/8/15
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Chairman of Department	Date

GEOMORPHIC RESTORATION AND STABILIZATION
OF A REACH OF STEVENS CREEK

by

Gretchen Kayser
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SENIOR DESIGN REPORT PROJECT

submitted to the
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in partial fulfillment of the requirements for the degree of
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GEOMORPHIC RESTORATION AND STABILIZATION OF A REACH OF STEVENS CREEK

Gretchen Kayser, Nick Roby, and Travis Giffen

Department of Civil Engineering
Santa Clara University, Spring 2015

Abstract

This project focuses on the restoration of a 460-foot reach of Stevens Creek in Santa Clara County, California. The new design will increase the stability of stream channels, riparian system functions, and fish passage while improving or maintaining the “level of service” based on geomorphic data. This project was completed in conjunction with the Santa Clara Valley Water District’s standards and design criteria. The proposed design is needed to increase the stability of stream banks and rectify stream bed incision. This will, in turn, provide greater flood protection to the residents of Santa Clara County. In addition to safety for the nearby community, the environment immediately surrounding Stevens Creek will reap benefits from a geomorphic stream design that will result in lower maintenance costs by promoting sediment balance throughout the creek. Lastly, the rehabilitated creek will allow native steelhead trout to continue to swim upstream.

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Introduction

The engineering code states that the primary responsibility of an engineer is to "hold paramount the safety, health and welfare of the public" (NSPE). This accountability is transferred to the public sector, namely water districts across the nation, in regards to flood control and environmental health. In order to combat potential flooding and environmental degradation, the Santa Clara Valley Water District (SCVWD) has passed measures to ensure the county's protection for years to come. In November of 2012 the Safe, Clean Water and Natural Flood Protection Program was passed by voters. Within this program is the District's Creek Restoration and Stabilization Program, Priority D.6, which will use geomorphic data to design and construct projects to increase the stability of eroding creek banks and help restore the natural functions of stream channels. The scope of this work includes areas of Stevens Creek, Uvas Creek and Comer Debris Basin on Calabasas Creek. The particular project in question focuses on a 400 ft reach (or section) of Stevens Creek located between Moffett Boulevard and E Middlefield Road in Santa Clara County, as shown below in Figures 1 and 2.

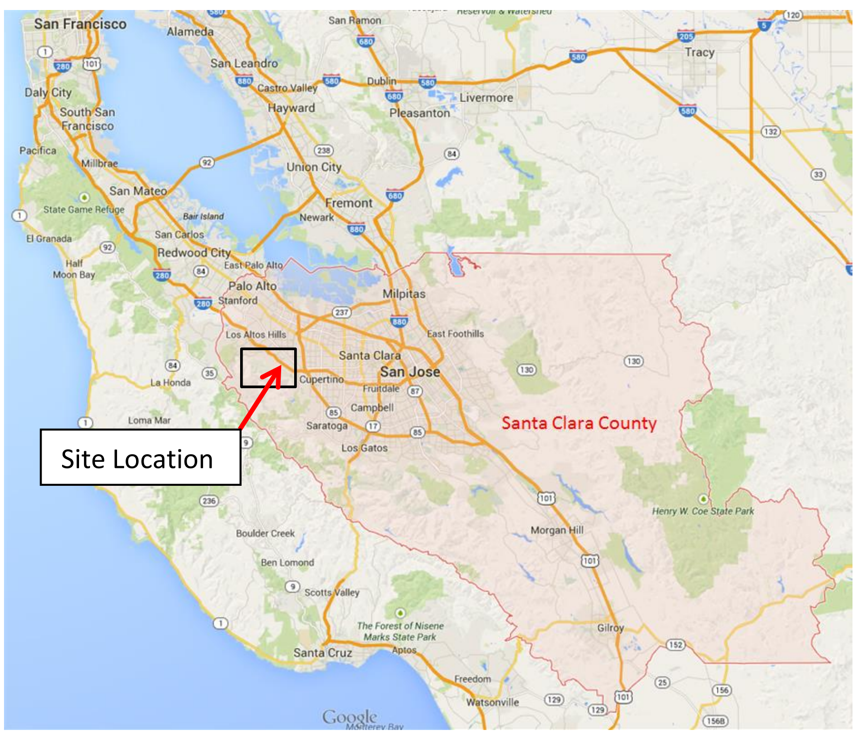

Figure 1: Area map displaying Santa Clara County

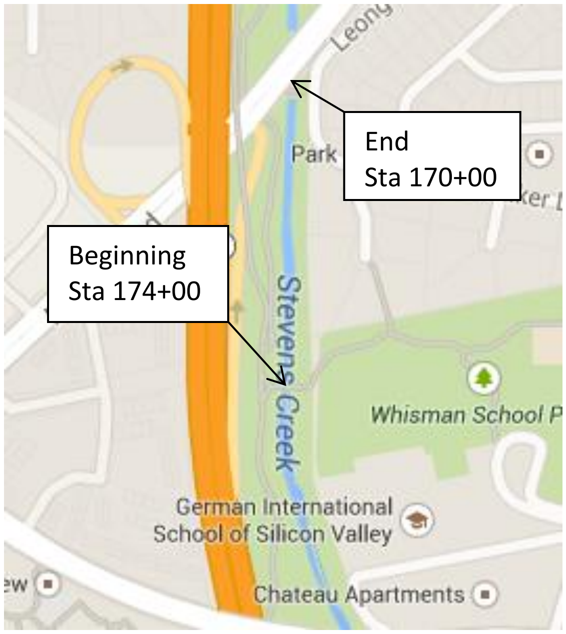

Figure 2: Area map displaying the reach of Stevens Creek to be redesigned.

This reach was selected because of its challenging design constraints and the high priority need to repair the fish passage and stabilize the bank. Our focus was to resolve these concerns while maintaining or increasing flood protection, in keeping with the District goal of restoring stability and stream function by preventing incision and promoting sediment balance throughout the watershed.

In meeting this objective, Project D.6 will benefit the Santa Clara County areas of Saratoga, Mountain View, Sunnyvale and Gilroy. The SCVWD is scheduled to start the planning phase of this project at the beginning of the fiscal year 2018 with project completion scheduled for fiscal year 2021. The $16 million dollar project will minimize erosion, sedimentary deposits, improve the general ecosystems of the creeks, and provide flood control.

Ethical Considerations

It is important to consider potential ethical issues that will impact or, that may develop within, the course of this project. The parties impacted by this project include the wildlife within the Stevens Creek ecosystem and the neighboring residents. In addition, there is a clear impact on the natural environment of Stevens Creek and the greater South Bay area.

One of the main goals for the project is to improve the habitat for the natural wildlife within the creek. A geomorphic design will ensure this environmental improvement while also mitigating against future flood hazards. However, problems may arise when construction is conducted and the design of this project must be mindful of this. Thus, the design must work to mitigate the negative effects of construction on the wildlife and environment. Ultimately, this project solves, rather than creates, most non-technical issues.

Since this project was developed with guidance from 16,000 residents and stakeholders of Santa Clara County and was voted on by the people of Santa Clara County, the moral aspect within the social justice element of this project is understood to be high. Lastly, there is no conflict of interest, and eminent domain will be avoided in the design. Any other ethical issues will be monitored in the subsequent design phase and will be enforced by following government regulations.

Design Criteria and Standards

Federal and State Standards and Criteria:

The restoration of any water body, such as Stevens Creek, must fulfill numerous procedures and design constraints. Table 1 shows an overview of all the federal and California state regulations involved with altering a body of water and the agency responsible for each.

Table 1: Federal and California State Regulations and Associated Regulatory Agency

Regulation	***Responsible Agency***
Federal Endangered Species Act	US Fish and Wildlife Service National Marine Fisheries Service
Federal Clean Water Act; Porter-Cologne Act, California Water Code	US Environmental Projection Agency US Army Corps of Engineers State Water Resources Control Board Regional Water Quality Control Boards
California Coastal Act	California Coastal Commission
Coastal Zone Act Reauthorization Amendments Section 6217	National Oceanic and Atmospheric Administration US Environmental Protection Agency
Fish and Game Code Section 1601 and 1603; California Endangered Species Act	California Department of Fish and Game
California Environmental Quality Act	Various state and local agencies
Erosion and Grading Ordinances, Development Standards, Habitat Conservation Plans, Local Coastal Plans and other local permits	County government

- **Federal and State** On the federal and state level many permitting requirements must be satisfied. The design plan titled Safe Clean Water Act Priority D.6 has already been approved by the public, although that is only the first step in procurement. Government agencies must permit the project in order for construction to be carried out.
- **State of California**. The California Department of Fish and Game (DFG) administers the Lake and Streambed Alteration (LSA) Program pursuant to the Fish and Game Code (FGC) sections 1600-1616.
  - FGC Section 1602 requires an entity to notify California Department of Fish and Game (DFG) of any proposed activity that will:
    - substantially divert or obstruct the natural flow of any river, stream or lake;
    - substantially change or use any material from the bed, channel, or bank of, any river, stream or lake; or
    - deposit or dispose or debris, waste, or other material containing crumbled, flaked or ground pavement where it may pass into any river, stream, or lake (CA).

- The notification requirement applies to work undertaken in or near any river, stream, or lake. If DFG determines that the activity described in a notification may substantially affect an existing fish or wildlife resource, DFG and the project owner enter into an LSA Agreement that includes reasonable measures necessary to protect the resource.
- DFG must comply with the California Environmental Quality Act (CEQA) before it can execute a LSA Agreement. Conveniently, a maintenance program through the SCVWD has already been permitted which includes blanket CEQA permitting.

The ultimate goal is to receive approval for project D.6 under the small-capital maintenance program. This will streamline the permitting processes although documents such as an Environmental Impact Report (EIR) will still need to be completed for the project separately.

District Standards

Since the project is overseen by the Santa Clara Valley Water District (SCVWD) it will comply with all district project procedures and protocol.

Design Criteria

Other criteria that must be met are that of design itself. There are four major project design criteria goals that must be addressed with this project.

- **Minimize erosion and incision that the creek currently experiences**. Erosion is the gradual destruction or diminution of something, in this case the creek banks, and incision can be explained as the undercutting of a creek's banks which leads to the straightening and steepening of a channel.
- **Maintain or improve the creek's current flood protection levels**. Currently the creek can maintain a 100 year flood occurrence flow, which is the estimated heaviest flow that will occur in 100 years, thus having a 1% chance of occurrence each year. However, with global warming this value might increase. Therefore, improved flood control rose in importance for our design.
- **Improve the overall ecosystem health of the creek**. By eliminating cracking and broken concrete structures from the creek and replacing these stabilizing entities with natural alternatives the ecosystem has a better chance to return to a more natural state.

- **Improve the fish passage through the reach.** The existing 3 ft drop at the beginning of the reach does not allow for fish passage, so our design process ensured compliance with fish passage criteria to allow for steelhead migration further upstream.

Creek Initial Conditions

The initial conditions of the selected reach of Steven's Creek were determined based on the data from both the existing HEC-RAS models provided by the SCVWD and our visual observations of the creek. As seen in Figure 3, much of the existing section consists of trapezoidal concrete channel.

Figure 3: Trapezoidal Concrete Section of Selected Reach

Because of the design criteria set forth by the SCVWD any modification to the existing concrete channels or structure will not be permitted, and therefore, all variables such as slope, Manning's n, and flow velocities associated with these sections had to remain constant.

A particular focus for design was the interface of the existing concrete channels and the natural earthen stream banks. As a result of the change in cross section at these areas, largescale incision, bank instability, and erosion are present. Figure 4 highlights one example of this existing problem.

Figure 4: Heavy Erosion Located at Concrete and Earthen Channel Interface

As you can see at the base of the channel in Figure 4, severe erosion issues are present that threaten the stability of the banks. This issue is most pronounced at the transition locations between concrete and earthen sections of the reach.

The third and final variation of preexisting cross section consists of a trapezoidal channel constructed using rip-rap. This section has noticeably more abundant natural vegetation and animal life as well as reduced erosion and bank stability issues. Figure 5 depicts the existing conditions present at this section of creek, a condition that the design plans to implement in the earthen section of the channel.

Figure 5: Existing Trapezoidal Channel Constructed Using Rip-Rap

The reach in Figure 2 that suffers erosion and bank instability has a hundred year flow capacity of 3,392 cfs, slope of 1%, and must withstand flow velocities of 7.93 ft/s. Additionally, an existing drop located at a crossing of the Hetch Hetchy Aqueduct creates a barrier to fish passage. An image of the drop is provided in Figure 6 below.

Figure 6: Hetch Hetchy Aqueduct Passage and Associated Drop a Barrier to Fish Passage

As shown in Figure 6, the existing drop represents a significant barrier to fish passage.

Description of Proposed Solutions

Alternative design configurations to increase bank stability, fish passage, and erosion resistance for the proposed section of creek took into account several different site-specific criteria. An outline of the proposed different alternative designs can be seen in Table 2.

Table 2: Advantages and Disadvantages of Alternative Designs

Design Alternative	Advantages	Disadvantages
Increase Channel Slope (Remove Drop) and Add Wing Dam	• Allows for Fish Passage • Environmentally Beneficial (Stabilizes Bank and decreases Incision) • Aesthetic	• More useful for larger sites, not useful for smaller creeks; our creek is too small for this solution • Higher comparative maintenance costs • Might decrease LOS too drastically
Add a Levee and Decrease Bank Slope and Increase Channel Slope (Remove Drop)	• Low Maintenance Costs • Less Erosion • Less Need for rip-rap • Higher Estimated LOS	• High Initial Cost • Periodic Mandatory Inspection of Levees • Possibility of Levee Failure • Incision issue still present
Maintain Drop (Maintain Small Slope) and Add Fish Ladder and Utilize rip-rap to Stabilize the Bank	• No Potential Increase in Velocity or Shear Stress • Allows for Fish Passage	• High Initial and Maintenance Cost of Fish Ladder • Not Best Alternative for Environment • Incision issue still present
Increase Channel Slope (Remove Drop) and Add Step Pools and Rip - Rap	• Allows for Fish Passage • Environmentally Beneficial • Aesthetic • Reduces Erosion and Incision	• Potential for Increased Velocity and Shear Stress

Overall, the decision process for the final design sought to identify a solution that addressed the needed criteria while minimizing any environmental impact. To fulfil this goal, the proposed solution needed to maintain close to natural stream characteristics while still addressing the bank stability, erosion, and fish passage issues of the existing site. Thus the final option in Table 1, Increase Channel Slope, which removes the hydraulic drop, adds step pools, and adds rip-rap, was chosen. The bank stability will be improved by adding rip-rap to the existing vertical creek banks. This will center the creek in the channel and prevent bank erosion and the resulting bank instability. The slope of the channel will be designed to maintain the existing flow velocity and decreasing incision, while allowing for proper fish passage. Additionally, appropriately sized boulders will be added to the creek bed to dissipate energy to prevent further erosion and incision.

Reference Reach: Blackberry Farm

The SCVWD completed the first stage of a restoration project of a reach of Steven's Creek upstream of current design area in 2009. This $1.39 million dollar restoration improved the existing earthen trapezoidal channel into a rip-rap reinforced step pool design. Figure 7 below illustrates the dramatic improvement in the health of riparian system of the creek.

Figure 7: Before and After the SCVWD Blackberry Farm Creek Restoration

Measurements were taken from the Blackberry Farm reference reach that included step pool length, channel width, and anchor boulder size in order to cross reference these values during the design process. The restored section at Blackberry Farm has experienced several high flow events since its completion in 2009 and has proven to be a resilient and self-maintaining design that also promoted a more robust ecosystem. For these reasons this section was closely referenced during the design process for the current project.

Step-Pool Design

The ultimate channel slope design was created within the two constraints upstream and downstream of our reach. Upstream of our site is the Hetch Hetchy aqueduct crossing, where a concrete channel exists creating a three foot hydraulic drop. Downstream is a concrete channel that feeds into two box culverts. The ultimate slope over this 463.5 foot span is 1%. With this information we decided that the appropriate amount of step pools for our reach would be 3 based on criteria set forth in the Reference Reach section above and Fish Passage section below. A step pool is a design feature in rehabilitated creeks that allows for steepening slopes with decreased incision, scour, and fish barriers. The features of a step pool are summarized by Figure 8. From there the height of our weir, the depth of our pool, the size of our boulders, and the length of our riffle were calculated. In doing so we were able to increase the overall slope within our reach while increasing fish passage and allowing the erosion and incision of the creek to be minimized. The design process and modelling for each aspect of the step pool are discussed below.

Figure 8: A Basic Step-Pool Profile

Major Issues Addressed: Fish Passage and Creek Weathering

The two major issues addressed in this Step-Pool design are fish navigation and creek weathering. The first of these issues considered in our design pertained to increasing fish navigation. In order to do so, as stated above, the elevation of the creek needed to be raised three feet on the upstream side of our reach. Our goal was to solve this issue within the 463.5ft of our reach, meaning the elevation on the downstream end was to stay at its existing level, therefore increasing the overall slope within our reach.

Our concern in doing so was that erosion and incision, which were already an issue in the existing condition, would increase along with the increase in slope. Thus the design had not only to solve the current weather problem, but also to account for the changes made to the creek's structure. When designing an open flow channel to reduce incision and erosion, the focus and concern is on the shear stress being delivered by the water under maximum capacity in comparison to the shear stress the creek structure is able to withstand. Shear stress is the force vector component parallel to the cross section, in this case the creek bed and banks. If the creek were to deliver a shear stress greater than what the bed and banks can withstand according to their material properties, the creek's structure will start to break down. Since shear stress is delivered to the boundaries of the creek through a decrease in the flow's velocity along the boundary, we used flow velocity to create our design, a well-accepted method in Civil Engineering practice. Ultimately the Step-Pool design accounted for the issues pertaining to shear stress and was verified that fish passage remained adequate across the designed reach.

Design Process

The first step in the design of the step-pool was to create its geometric structure. As mentioned above, the design contains three riffle pool systems, each one containing its own drop. The reason for three is to allow for the best chance for fish passage. Two riffle-pools would have led to a riffle length that was too long for the fish to navigate, while four may have led to riffle that were too steep. In the current design, the system has a 4.6 total drop which is about 1.5ft for each riffle-pool system. Using geometric data, flow data and equations 1 and 2, the calculations resulted in a scour depth of 1.43ft and a minimum pool length of 35.1 feet. Accounting for fish passage, we set the riffle lengths to 104.5ft and the pool lengths to 50ft, as calculated with the following equations:

$$\frac{\text{Scour Depth}}{W} = -0.0118 + 1.394\frac{H}{W} + 5.514\frac{S_0 q_{25}}{\sqrt{g}W^{3/2}} \qquad \text{(equation 1)}$$

$$\frac{\text{Pool Length}}{W} = 0.409 + 4.211\frac{H}{W} + 87.341\frac{S_0 q_{25}}{\sqrt{g}W^{3/2}} \qquad \text{(equation 2)}$$

HEC-RAS Modeling

Design Results

This design was input into a design program called HEC-RAS, which stands for Hydrologic Engineering Centers River Analysis System. It is a one dimensional flow program that, in our case, takes geometric survey data, as well as natural flow data, and creates a model of the programed creek section. Once our design was modeled on HEC-RAS, the program revealed the maximum velocity in the system. This information, in conjunction with the knowledge garnered from our reference creek at Blackberry Farms, allowed for the design of the boulders that will be placed in the creek to anchor the creek structure. These interlocking boulders, buried approximately two-thirds of the way, will anchor the bank and provide for stability. The SCVWD approach to these design calculations was followed as shown in Appendix B. This resulted in backing no. 1 sized boulders along the creek banks and 5x2x3 foot anchoring boulders to be used at the step-pools as seen in Figure 9 below.

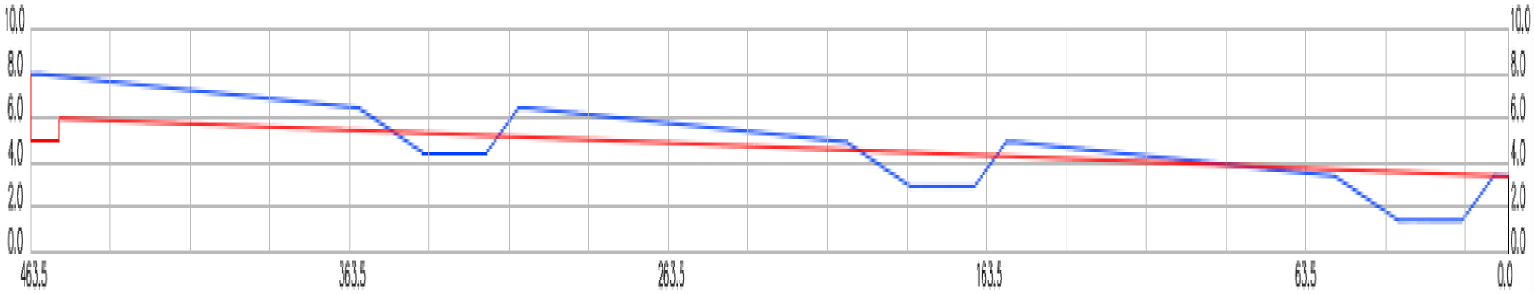

Figure 9: Existing and Proposed Profile of the Reach

The AutoCAD drawing as seen in Figure 9 illustrates the existing profile view of the reach in red and the proposed step-pool design profile in blue with units of feet.

Further use of HEC-RAS Modelling

The next step to combat these issues was to determine the current shear stress along the banks and to check the designs performance. To do this HEC-RAS was utilized in order to determine the maximum flow conditions, the condition in which the creek experiences the largest shear stress, or in our case, largest velocity. This data was later used to design the bank stability and ensure fish passage. The results are described in the Fish Passage section below.

Fish Passage Design

As previously mentioned fish passage was a vital component to this creek restoration design. Earlier in 2015 the SCVWD determined fish passage criteria specifically for Stevens Creek while working on a project located further upstream of the project reach. This criterion was based on fish passage requirements for the native steelhead trout. Utilizing the HEC-RAS model with the input of two year recurrence interval flows (a minimum of 16 cfs and a maximum of 70 cfs) three major standards for fish passage were checked. These required criteria included maximum average water velocity, minimum flow depth and maximum drop the steelhead can withstand. Table 3 below shows the required values in black and the model output results in red.

Table 3: Steelhead Fish Passage Requirements for Stevens Creek, CA

Maximum Average Water Velocity and Minimum Depth of Flow			
Species and Life Stage	**Maximum Average Water Velocity (fps)**	**Minimum Flow Depth (ft)**	**Maximum Drop (ft)**
Adult steelhead	4.0* 3.92	0.8-1.0 2.0	1 0-1.0

*Maximum average water velocity is dependent on culvert or in this case low flow channel length (Appendix F).

It can be seen that the all the design requirements were met, therefore proving that the design is conducive to fish passage.

Rip-Rap Gradation

Rip-rap is a term used to describe loose stone used to form a foundation or to protect shoreline structures. After consulting with SCVWD engineers and integrating our observations from the reference reach of Blackberry Farms, we decided to 'rock' the entire channel. This method protects the banks from high velocity flows while still maintaining the most natural creek environment possible. The finished result of such a process will look similar to the creek bank shown in Figure 9 below.

Figure 10: Rocked Channel Design Example

The type of rip-rap that will be used in the Stevens Creek design can be separated into four categories. The majority of the rip-rap used in the restoration, which will cover the creek banks and riffle sections, was determined using the California bank and shore rock slope protection design specification. This is a Cal Trans method approved by the SCVWD. This process uses Equation 3 to determine the minimum stone weight, 6.58 lbs.

$$W = \frac{0.00002 * (0.67 * v)^6 * SG}{(SG-1)^3 * (\sin(r-a))^3} \qquad \text{(equation 3)}$$

Next, the Outside Layer Design for Caltrans (2000) Table, found in the Appendix E, was utilized to determine the gradation of rip-rap. Based on this table, Backing No.1 rock was selected. This rock is approximately 1 ft in diameter and will be placed in a layer with a minimum thickness of 1.8 ft around the creek banks and riffle sections. This will allow for greater rock stabilization and protection.

The second type of rip-rap is that placed at the bottom of the scour pools. To determine the size of these rocks the SCVWD approved Isbash method was utilized. The Isbash equation shown below by Equation 4 determined the average diameter of the rock to be 5 inches.

$$D = \frac{v^2}{2*g*C^2*(SG-1)} \qquad \text{(equation 4)}$$

These rocks can be smaller than those previously selected because the pools, which serve as resting places for the fish, will experience lower velocities. The minimum layer of thickness for this rip-rap is 0.75 feet.

The third type of rock sized were the anchor rocks, or boulders, placed at the beginning of the pools and end of the riffles. These boulders will be the largest rocks utilized and will experience the greatest force from the rushing water. Based on our reference reach of Blackberry Farms and consultation with Santa Clara Valley Water District engineers, we determined that 2x3x5 ft boulders should be used. These boulders will be buried two thirds of the way in the ground (with the 5 ft dimension vertically placed) to ensure stability.

Lastly, in addition to these three types of rip-rap a small amount of gravel will be placed in the creek to fill in the gaps between the larger rocks and create a more natural environment.

Cost Estimate

The cost analysis for the restoration of Stevens' Creek was completed in two stages. First the raw material costs were determined using RSMeans Building Construction Cost Data book. The second stage consisted of determining the manpower and equipment needed for the construction and their associated prices.

The three basic materials that need to be delivered to the site for construction are the rip-rap for bank stability, the larger anchoring boulders for the step pools, and a polypropylene stabilization fabric placed on the bank surface prior to positioning the rip-rap. Each of these items was specified in the RSMeans manual and their price, including delivery to the site and the general contractor's markup, and was calculated based on the square-yard surface area of our reach. An additional cost to be considered is the disposal fee charged by the local landfill for dumping excess soil from the excavation. This price was provided by per cubic yard from the Zanker Recycling Center. The total resulting price was multiplied by a factor of 1.19 to find a more accurate estimate that takes into account a multiplied inflation factor 1.03 and a location adjustment of 1.15. These values are given in the City Cost Index Table of the RSMeans manual. The final cost of the raw materials for construction was determined to be $487,620.58.

The estimation of labor and machinery rental costs were also determined using the RSMeans reference. First the group estimated the amount of labor, time, and machinery required for construction based on consultation with the SCVWD. It was determined that construction would require a backhoe, dump truck, crawler crane, and clamshell bucket. This machinery will be used to place the riprap and alter the banks and form the step-pools as specified in the design. Along with the operators of these pieces of machinery three additional laborers would be needed to help with the placement of the stabilization fabric and rip-rap. All of these requirements were fulfilled by combining Crew B-17, Crew B-12G, and Crew B-34-B. The daily rate, including contractor's markup, location index, and inflation was determined from the crew tables in RSMeans. The estimated project duration of 36 days was determined after consultation with the SCVWD. The total daily rate and project duration were multiplied to determine the cost of labor and machinery rental. This was determined to be $185,745.57. Finally, the total cost of the project was calculated by adding the raw material total with the labor and machinery amount and was found to be $673,478.91.

Conclusion

This project utilized information and models provided by the SCVWD to obtain the existing design variable for the reach. Erosion and stability issues with the existing sections were identified by visual observation and incorporated in the HEC-RAS models used in developing the design solution. Flow capacity values and soil properties were also obtained using models, information, and equations provided by the SCVWD. HEC-RAS is a free and commonly used software program developed by the United States Army Corps of Engineers to model the hydraulics of 1-dimensional water flow through channels and natural rivers.

Utilizing the HEC-RAS modeling software, an iterative design process was developed in which the design variables including slope, Manning's n, and flow capacity were altered in an effort to design an optimum compromise between capacity, reduced erosion, increased fish passage, and improved environmental conditions. Because existing concrete structures could not be altered per the SCVWD's guidelines, the primary focus was on methods to reduce erosion of the earthen sections of channel as well as the hydraulic drop to improve fish passage at the location of the Hetch Hetchy Aqueduct crossing.

The reduction of erosion and increase bank stability was ultimately accomplished by utilizing a rip-rap based trapezoidal channel design in place of the existing earthen sections of the channel. rip-rap structure was calculated via a spreadsheet provided by the SCVWD. Increased fish passage was accomplished by increasing the slope of the channel downstream of Hetch Hetchy Aqueduct crossing, which resulted in a reduced hydraulic drop at this location. To counter the increased flow velocities associated with increasing channel slope, step pools and large rip-rap were utilized to dissipate the energy while promoting a more hospital environment for local wildlife and vegetation.

Appendix A
Blackberry Farm Reach

5’ Anchor Boulders Secure End of Step-pool

Interlocking Anchor Boulders Allow Fish to Swim Though Gaps in Rock-Promotes Fish Passage

Visible Rip-Rap Embedded in Banks Increases Bank Stability and Provides Erosion Protection

Appendix B
Step-Pool and Rip-Rap Gradation Calculation




The Santa Clara University student group designed a 463 ft reach of Stevens Creek as part of the Project Priority D.6, which included remediating in-channel concrete structures, improving fish passage and repairing bank erosion. In the creek there was a 3 ft concrete drop that was identified as a fish passage barrier. It was determined that the best way to remediate this would be to replace this barrier with step pool structures.

Using the design manual guidelines, the number, pool depth, drop height, and length of the weir and pool structures were determined, as well as the riprap size.

Given:

- Floodplain width, $W_{fp} = 0$ (incised channel)
- Bankfull channel width, $W = 66.0\text{ ft}$
- $Q_{1\%} = Q_{100} = 3{,}392\text{ cfs}$
- $Q_{4\%} = Q_{25} = 1{,}668\text{ cfs}$
- Channel bed slope, $S = 0.010\text{ ft/ft}$
- Acceleration due to gravity, $g = 32.2\text{ ft}/s^2$

Design procedures are from Design Manual, Page 3-17.

(1) Determine the design flow for the weir using Eq. (3-1) and compute the corresponding average velocity in the channel using HEC-RAS.

$$\frac{W_{fp}}{W} = \frac{0 \text{ ft}}{66.0 \text{ ft}} = 0 < 1$$

$$\therefore Q_{\text{design}} = Q_{1\%} \text{ or } Q_{\text{Project Design}} \rightarrow \text{ use } Q_{1\%} \text{ 3,392 cfs}$$

The corresponding creek reach average velocity from the HEC-RAS design model: V=7.93 ft/s

(2) Design the scour pool as described in Section 5.4.5.1.

(a) Determine design flow

As stated in the step (1) above, use $Q_{1\%} = 3{,}392$ cfs *as the design flow for sizing the rocks of the scour hole.*

In addition to this design flow, the design manual calls for the unit 25-year discharge, q_{25}*, determined by:*

$$q_{25} = \frac{Q_{25}}{W}$$

$$q_{25} = \frac{1{,}668 \text{ cfs}}{66 \text{ ft}}$$

$$q_{25} = 25.3 \text{ cfs/ft}$$

Where:

- W = bankfull width
- Q_{25} = discharge of 25-year return interval

(b) Determine scour depth

$$\frac{\text{Scour Depth}}{W} = -0.0118 + 1.394\frac{H}{W} + 5.514\frac{S_0 q_{25}}{\sqrt{g}W^{3/2}}$$

$$\frac{\text{Scour Depth}}{W} = -0.0118 + 1.394\frac{1.5 \text{ ft}}{66 \text{ ft}} + 5.514\frac{(0.01 \text{ ft/ft})(25.3 \text{ cfs/ft})}{\sqrt{32.2 \text{ ft/sec}^2}\left(66 \text{ ft}^{3/2}\right)}$$

$$\frac{\text{Scour Depth}}{W} = 0.02$$

$$\text{Scour Depth} = 0.02W$$

$$\text{Scour Depth} = 0.02(66\text{ft})$$

$$\text{Scour Depth} = 1.34 \text{ ft}$$

Where:

- H = drop or weir height
  From Thomas, et al (2000):
  > *Step height is the independent variable in the design process. When constructing step-pool structures for channel stabilization purposes, the drop height will reflect the elevation loss that must be accommodated to stabilize the channel while meeting the low-drop criteria up to the two-year discharge. Similarly, the step height may be chosen to determine habitat value in the downstream pool.*

  If fish passage is not an issue, H can be set at 1 to 2 feet. If fish passage is an issue, it will be necessary to consult with a fisheries biologist. Also, the California Salmonid Stream Habitat Restoration Manual published by the California Department of Fish and Game may be useful: http://www.dfg.ca.gov/fish/REsources/HabitatManual.asp
  It may take a few tries to determine the appropriate value for H that provides the right scour depth and pool length.
- S_0 = channel slope
- g = gravitational acceleration, 32.2 ft/sec^2

(c) Determine pool length (L_2):

$$\frac{L_2}{W} = 0.409 + 4.211\frac{H}{W} + 87.341\frac{S_0 q_{25}}{\sqrt{g}W^{3/2}}$$

$$\frac{L_2}{W} = 0.409 + 4.211\frac{1.5\text{ ft}}{66\text{ ft}} + 87.341\frac{(0.01\text{ ft/ft})(25.3\text{ cfs/ft})}{\sqrt{32.2\text{ ft/sec}^2}(66^{3/2})}$$

$$\frac{L_2}{W} = 0.5$$

$$L_2 = 0.5W$$

$$L_2 = 0.5(66\text{ ft})$$

$$L_2 = 33.8\text{ ft}$$

(d) Determine maximum pool width (B_3):

$$B_3 = 1.2W$$

$$B_3 = 1.2(66\text{ ft})$$

$$B_3 = 79.2\text{ft}$$

(3) Determine gradation for the channel bed and bank protection described in Section 5.2.1.1.

(a) *The rock weight is determined using Eq. (5-5):*

$$W = \frac{0.00002 * (0.67 * v)^6 * SG}{(SG-1)^3 * (\sin(r-a))^3} = \frac{0.00002 * (0.67 * 7.93)^6 * 2.65}{(2.65-1)^3 * (\sin(70-55))^3} = 6.58lb$$

Where:

- $W =$ theoetical minimum rock weight, lb
- $V =$ velocity to which bank is exposed : (0.67 times the channel average velocity for parallel flow)
- $r = 70$, angle of repose for randomly placed rubble
- $a =$ angle of outside slope with horizontal, degrees

(b) *The rock size is determined using Table 5-2:*

Table 5-2
Outside Layer Design for Caltrans [2000]

STANDARD Rock SIZE or Rock MASS or Rock WEIGHT		GRADING OF ROCK SLOPE PROTECTION **PERCENTAGE LARGER THAN**											
		RSP-Classes [A]											
		Method A Placement					Method B Placement						
		RSP-Classes other than Backing									Backing No.		
US unit		8 ton	4 ton	2 ton	1 ton	1/2 ton	1 ton	1/2 ton	1/4 ton	Light	1 [B]	2	3
	SI unit	8 T	4 T	2 T	1 T	1/2 T	1 T	1/2 T	1/4 T	Light	1 [B]	2	3
16 ton	14.5 tonne	0-5											
8 ton	7.25 tonne	50-100	0-5										
4 ton	3.6 tonne	95-100	50-100	0-5									
2 ton	1.8 tonne		95-100	50-100	0-5		0-5						
1 ton	900 kg			95-100	50-100	0-5	50-100	0-5					
1/2 ton	450 kg				95-100	50-100	-----	50-100	0-5				
1/4 ton	220 kg					95-100	95-100	-----	50-100	0-5			
200 lb	90 kg							95-100	-----	50-100	0-5		
75 lb	34 kg								95-100	-----	50-100	0-5	
25 lb	11 kg									95-100	90-100	25-75	0-5
5 lb	2.2 kg											90-100	25-75
1 lb	0.4 kg												90-100

[A] US customary names (units) of RSP-Classes listed above SI names, example US is "2 ton" metric is "2 T".
[B] "Facing" has same gradation as "Backing No. 1". To conserve space "Facing" is not shown .

Where the rock weight class is conservatively increased to the next highest size (25 lb) and the gradation class of 90-100 is chosen leading to Backing No. 1 [B] rock chosen.

(4) Determine gradation for the scour pool described in Section 5.2.1.1.

(c) *The rock size is determined using Eq. (5-8):*

$$D = \frac{v^2}{2 * g * C^2 * (SG-1)} = \frac{7.93^2}{2 * 32.2 * 1.20^2 * (2.65-1)} = 0.41ft = 4.93\ in$$

Where:

- D = stone size (ft)
- V = velocity to which bank is exposed
- g = gravitational acceleration (ft/s^2)
- C = Ishbash coefficient

Spreadsheet Used to Design

	Factor	Variable	Value	Units	Formula
	Acceleration Due to Gravity	g	**32.2**	ft/s^2	
	Bankfull Width	W	**66.0**	ft	
	Weir Height	H	**1.5**	ft	
	Average Channel Slope	S_0	**0.010**	ft/ft	
	Design Discharge	Q_{Design}	**3,392**	cfs	
	25-Year Discharge	Q_{25}	**1,668**	cfs	
1	Design Unit Discharge for Weir	q_{Design}	**51.4**	cfs/ft	$q_{Design} = \frac{Q_{Design}}{W}$
	25-Year Unit Discharge for Weir	q_{25}	**25.3**	cfs/ft	$q_{25} = \frac{Q_{25}}{W}$
2	$\frac{\text{Scour Depth}}{W}$	$\frac{S}{W}$	**0.02**		$\frac{\text{Scour Depth}}{W} = -0.0118 + 1.394\frac{H}{W} + 5.514\frac{S_0 q_{25}}{\sqrt{g}W^{3/2}}$
	Scour Depth	S	**1.34**	ft	
3	$\frac{\text{Pool Length}}{W}$	$\frac{L_2}{W}$	**0.5**		$\frac{\text{Pool Length}}{W} = 0.409 + 4.211\frac{H}{W} + 87.341\frac{S_0 q_{25}}{\sqrt{g}W^{3/2}}$
	Minimum Pool Length	L_2	**33.8**	ft	
4	Maximum Pool Width	B_3	**79.2**	ft	Maximum Pool Width = 1.20 * Weir Width
	Maximum Pool Velocity	v	**7.9**	ft/s	
	Isbash coefficient	C	**1.20**		
	Specific gravity of rock	G_s	2.65		
	Minimum Boulder Size, Pool	$D_{50\ pool}$	0.41	ft	$D_{50} = \frac{v^2}{2gC^2(G_s - 1)}$

Appendix C
Cost Estimate

Work Item	Quantity	Unit	Unit Cost	Total O&P	Time Factor	Location Factor	Total	Vendor/Source	Subtotal	Notes
Materials										
Materials									**$487,620.58**	
Stabilization Fabric	2916.10	S.Y.	1.26	1.43	1.03	1.15	$ 4,939.39	RSMeans 2012 pg.611		
General Rip-Rap-18' minimum thickness machine placed	2,916	S.Y.	72.9	97.50	1.03	1.15	$ 336,776.36	RSMeans 2012 pg. 600		
Anchor Rocks	3	Ton	26.66	29.50	1.03	1.15	$ 104.83	RSMeans 2012 pg. 600		
Soil Disposal Fee	2,916	Cubic Yard	50	n.a	n.a	n.a	$ 145,800.00	http://www.zankerrecycling.com/zanker-facilities/disposal-fees/		
Crew Teams and Machinery										
Crew/Machinery Required									**$185,858.33**	
B-17	36	Day	1893.40	2593.34	1.03	1.15	$ 110,585.20	RSMeans 2012 pg.687		truck, 2 ops, 2 labors, backhoe
B-12G	36	Day	1348.10	1762.59	1.03	1.15	$ 75,160.36	RS Means 2012 pg. 685		Crane, Bucket, Op, laborer
B-34-B (1066)	16	180 Cubic Yards	4.82	5.95	1.03	1.15	112.7644	RS Means 2012 pg. 591		10 miles haul at 35 mph
Total Cost:			**Area Reach (SY):**							
$673,478.91										
							ft^2	yrd^2		
			Length:	463.50		bottom area:	9270	1030.00		
			Bottom Width	20		Bank area	16974.87066	1886.10		
			Slopes 15'drop @							
			50 degree angle			**Total Area:**	**2916.10**			
						Volume of Cut:	2916.10			
			bank length	18.31162						

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CHAPTER 3. GRADE CONTROL STRUCTURE DESIGN

3.1 INTRODUCTION

The term grade control may be broadly applied to any alteration in the watershed which provides stability to the streambed. The most common method of establishing grade control is the construction of in-channel grade control structures. Other methods include control of sediment supply and/or surface runoff to the streambed.

There are two basic types of grade control structures. One is referred to, by Biedenharn and Hubbard [2001], as a bed control structure, as it is designed to provide a hard point in the streambed that is capable of resisting the erosive forces of the flow in a degradational reach. The structure is built at grade and does not change the upstream or downstream flow conditions. The other type is referred to as a hydraulic control structure, as it is designed to reduce the energy slope to the point that the stream is no longer capable of scouring the bed. The structure is built above grade and will cause a backwater effect to the upstream flow. The difference of these two types of structures will be illustrated in the next section.

The design considerations for these two types of structures will be described first. Because of the District's intent to provide green design, materials such as concrete and gabions will not be used unless it's necessary for safety and constructability reasons. Because of our commitment to provide fish passage whenever possible, large drop structures which are fish barriers will not be considered. The design procedures that follow will provide a step by step guide to the analysis and design of grade control structures that meet our design requirements. The type of concrete drop structure that may be used at dam outlets will be discussed in Chapter 5.

3.2 DESIGN CONSIDERATIONS

The bed control structure may be made of rocks or logs anchored into the channel bed. Since its top elevation is made to level with the bed, the transition needs to be smooth to avoid pitting or flow induced erosion. The hydraulic control structure will protrude from the bed, raise the upstream water level and will need a plunge pool downstream to dissipate the extra potential energy. The structure is functionally a drop structure made of logs or rocks.

Design considerations for grade control structures include determination of the type, location, spacing of structures, and detailed design of the structures themselves. These considerations cover hydraulics, geomorphology, geotechnical, construction, maintenance and operation, and environmental impacts. Through these considerations, the design of grade control structures is performed. These considerations are explained in the following.

3.2.1 Design Flow

This is a design criterion that has not been made clear in the literatures and is worthwhile to clarify the concepts behind it.

A grade control structure is usually made as part of the bankfull channel geometry. Since we have established that the bankfull flow is the channel-forming flow that shapes the bankfull channel, intuitively one would design a grade control structure using the bankfull discharge, as suggested by *California Salmonid Stream Habitat Restoration Manual* [California 2006]. This approach is appropriate only if the project reach is composed of a bankfull channel and <u>wide</u>

floodplains. Calculations of channel-bottom shear stress (Figure 9-12b) have shown that the channel bottom will experience the maximum shear stress at the bankfull flow, when wide floodplains are present. Higher flows will result in reduction of the energy slope and bottom shear in the bankfull channel.

When the floodplain size is limited, or not present at all as in an incised channel, the bottom shear will continue to rise with flow, as shown in Figure 9-12b(3). In that case, which design flow to use becomes a question of risk management. The higher is the design flow, the higher the construction cost will be, but the lower the risk of failure will be.

Hence, the design flow for a project should be determined based on the channel geometry in the project reach. We have simplified the selection to the following approach:

$$\begin{aligned} \frac{W_{fp}}{W} &\geq 1 \qquad\qquad Q_{design} = 1.5Q_{bf} \\ \frac{W_{fp}}{W} &< 1 \qquad\qquad Q_{design} = Q_{1\%}\ or\ Q_{Project\ design} \end{aligned} \tag{3-1}$$

where W_{fp} is the floodplain width and W is the bankfull channel width as defined in Figure 9-12b(1).

The grade control structure should not suffer significant damage under the 1% flow, or project design flow, nor should it cause significant damage to the upstream or downstream reaches. Hence, if the Q_{design} is less than $Q_{1\%}$, the project should still check the integrity of the grade control structure using the latter.

For grade control weirs, the design flow is used to size the rock of the weir and design the riffle blanket for both log and rock weirs. This design flow is separate from the fish-passage flow which will be discussed in Chapter 5.

3.2.2 Hydraulic Considerations

The hydraulic siting of grade control structures is a critical element of the design process, particularly when a series of structures is planned. It involves the determination of the equilibrium slope, which has been discussed in Sections 9.1.2 and 2.2.1. The intent is to install the grade control structures to allow the existing slope to gradually evolve into the equilibrium slope which will be stable so long as the flow and sediment conditions remain.

Heede and Mulich [1973] suggested that the optimum spacing of grade control structures is such that the upstream structure does not interfere with the deposition zone of the next downstream structure. As shown in Figure 3-1, the desirable spacing (X) can be determined by extending a line from the top of the downstream structure at a slope equal to the equilibrium slope (S_f) until it intersects the existing slope (S_o).

The selection of a grade control structure may also depend on the geomorphic state of a stream. In a channel with well developed bankfull and floodplain geometry, it will be straightforward to fit a rock weir (Section 3.3.2) into the cross-section. However, a rock weir will be cumbersome to key into an incised channel, and an at-grade rock bench or log bottom may be used to minimize impact to the channel.

3.2.4 Geotechnical Considerations

The stability of channel banks will affect selection and design of grade control structures. When the critical bank height (Section 9.4) is exceeded due to degradation, bank instability may be widespread throughout the reach, rather than restricted to the outside banks in channel bends. For those cases, bank erosion protection may not be feasible, and grade control is more appropriate.

Also for banks that are near critical height, the grade control structure should be designed to reinforce stability by increasing toe protection and reducing bank slope. In most cases, the geotechnical data on soil material and bank stability are required to design grade control and bank protection structures.

3.2.5 Flood-Control Considerations

Channel improvements for stability reasons should always try to minimize impact to flood capacity. This requires that the grade control structures not encroach into the flow area. This is not an issue for bed control structures, but is a valid consideration for hydraulic control structures. The selection of a particular design should consider and quantify impacts to flow capacity and water level in the reach.

As a minimum the water surface profiles for the bankfull flow and maximum design flow should be examined to determine the effect of the grade control structure.

3.2.6 Environmental and Ecological Considerations

Sometimes at the location of a grade control structure determined from hydraulic calculations there are protected species of vegetation or wildlife such that construction of the grade control structure will incur significant environmental impact. Under these circumstances, it may be advisable to relocate the structure a short distance downstream or upstream to avoid or minimize environmental impacts. A field investigation is required after initial hydraulic calculations to determine if the site condition is feasible or optimal for locating a grade control structure.

Given our mission to preserve and protect the environment, fish passage is also an implicit assumption in our flood protection work. This assumption excludes large drop structures from being considered in design (see Section 3.3.2 for more detail). It also precludes concrete, gabion, and sackcrete from being used as materials of the grade control structure. Only the environmentally friendly materials such as gravels, rocks and tree logs are considered for grade-control purpose in this manual.

3.3 DESIGN PROCEDURE

The following procedure describes the sequence of technical analyses that are required to take place to produce the data needed for grade control structure design. It is assumed that land survey, basic water surface profile analysis (HEC-RAS, Sections 2.3.4 and 2.4.1) and sediment material characterization (Section 10.1) have been completed. Some of the analyses described below have already been discussed in Chapters 10 and 2. They are included here to address the relations to grade control structure design.

3.3.1 Sediment Transport Analysis

Since the purpose of grade control structures is to maintain or create a stable channel, the design has to prove a balance in sediment transport, i.e., sediment-in equal to sediment-out. To achieve this end, we need to determine the transport capacity, annual sediment yield, and equilibrium slope.

If the project reach is short and regular, or may be represented with a uniform slope and a constant cross-sectional geometry, use SAM for these calculations, as described in Section 2.4. The procedure is as follows:

(1) Develop a representative cross-section, longitudinal slope and sediment characteristics for the reach immediately upstream from the project reach.
(2) Calculate the transport capacity and sediment yield for this upstream reach using the methods described in Section 2.4.
(3) Develop a representative cross-section, longitudinal slope and sediment characteristics for the project reach.
(4) Calculate the transport capacity and sediment yield for the project reach.
(5) Compare results of Steps (2) and (4) and adjust the longitudinal slope of the project reach until the sediment yield from the project reach matches that of the upstream. This is the equilibrium slope.

Take this equilibrium slope and go to the next section to determine the need for grade control structures.

3.3.2 Hydraulic Siting Calculation

By comparing the equilibrium slope and the existing slope using Eq. (3-2) and (3-3), we can determine the need for grade control structures. The allowable drop height in most cases is defined based on the type of fish whose passage the design must accommodate. Both California Department of Fish and Game [2002] and NOAA [2001] require the maximum drop to be less than 1 ft for adult anadromous salmonids and 6 in. for juvenile salmonids. If the drop height exceeds this limit, opening may be considered through the drop structure. There are also requirement for the depth of the plunge pool which will be discussed in Chapter 5.

Hence, the procedure of hydraulic calculations is as follows:

(1) Develop the maximum drop height (h) based on fish passage and other criteria
(2) Use Eq. (3-2) to compute the required drop height (H)
(3) Use Eq. (3-3(to determine the number of drops (N) required and the spacing
(4) Layout the locations of drop structures on an aerial photograph

3.3.3 Field Data Collection

With tentative locations of the grade control structures determined, we are ready to go to the field for data collection. Bring a copy of the aerial photograph with tentative locations of the grade control structures marked on it, tape measure, and a note pad.

(1) Examine each of the tentative sites for constructability. The considerations include type of soil, vegetation that will be impacted by construction, channel geometry and bank stability, access for equipment and personnel, and construction lay-down and staging areas. If possible, locate grade control structures in straight channel sections to avoid nonuniform flow distributions intrinsic to channel bends.

(2) If a site is not suitable for construction, walk upstream and downstream to select another location in the close vicinity.

(3) After all locations are examined and confirmed, determine if additional geotechnical investigation should be conducted to assess bank stability and soil conditions. If the bank slope is steep and the terrace height is significant, there may be a potential of bank failure during excavation. Have a geotechnical engineer review the situation and make recommendations.

3.3.4 Type of Grade Control

With the location of the grade control structures identified, we can now determine which type of grade control best fits the project reach.

Generally there are two situations where grade control structures are needed. The first is when channel degradation has occurred or is in the processing of occurring, and we wish to install grade control structures to maintain the existing or future stable invert slope. The second is when degradation has generated a large invert drop at a specific location, e.g., a culvert or bridge crossing where the hardscape has stopped the degradation, and we wish to install grade control structures in the reach downstream from the drop to replace the large drop with several small drops.

Bed Control Structure

In the first situation, since the grade we are trying to control is at or lower than the existing grade, the bed-control type of structure is better suited for this application. This may be achieved by constructing a rock or log structure at grade. The design is provided in Section 3.4. No additional sediment transport analysis is required before design can proceed.

Hydraulic Control Structure

In the second situation, we will be trying to restore channel invert to a previous, higher elevation. The hydraulic-control type of structure will suit this purpose. To determine effect of sediment trapping on downstream reaches, and to determine timing of installation to minimize impact, the following analysis shall be performed:

(1) Construct a HEC-6 model to include the grade control project reach, an upstream reach to provide sediment inflow boundary condition, and a downstream reach long enough to extend into the depositional region, if this model has not been developed already.

(2) Using historical flow and invert profile data, calibrate the HEC-6 model.

(3) Develop flow record to simulate future creek flow condition.

(4) Run the HEC-6 model staging the installation of grade control structures to determine the optimal installation schedule and impact to the downstream.

Through this analysis the temporal and physical effects of grade control structures are determined. Now we can perform detail design of the grade control structures.

3.4 GRADE CONTROL STRUCTURE DESIGN

The main consideration in designing a grade control structure is the site cross-sectional geometry. In incised channels the flow area is usually restricted by steep banks. This situation requires a design that minimizes encroachment into the flow area. On the other hand, in channels with floodplains, a grade control structure may be integrated with the bankfull geometry without exacerbating flood conditions. Several different designs that can meet these considerations of the Santa Clara Valley are described below.

3.4.1 Incised Channel

If the stream is seriously incised, with high and steep banks and a terrace that has lost touch with the normal flow regime, the design should strive to minimize impact to the steep banks, but at the same time minimize encroachment into the flow area. These requirements limit the selection to simple log weirs and rock weirs as described below.

3.4.1.1 Log Weir

Using logs salvaged from the local watershed may best fulfill the purpose of grade control and habitat restoration in an incised environment. Logs have been used successfully for toe protection at SCVWD in the past, but relatively little experience exists for grade control. The design below was developed based on our own analysis and guidelines provided in the *California Salmonid Stream Habitat Restoration Manual* [California 2006] and *Integrated Streambank Protection Guidelines* [Washington State 2003].

A log weir is a drop structure made of logs or logs in combination with rocks. It may be a straight weir, downstream-V weir, upstream-V weir or diagonal weir. Sketches of these weirs are illustrated in Figure 3-4. More details on the design will follow later. A straight weir is perpendicular to the direction of flow, usually installed in a riffle section with flow uniformly distributed across the channel. A downstream-V weir has the tip of the V pointing in the downstream direction. Since it forces flow toward the banks, it is effective in dissipating energy, but should only be used in areas of stable banks that can withstand flow impact and shear accompanying such a design. The upstream-V weir, on the other hand, directs the flow toward the center and can develop a scour pool that enhances the aquatic habitat. A diagonal weir with a lower upstream end helps directing flow away from the bank of the downstream end. Designs of these log weirs are similar, and that is why only the straight-log-weir design is provided in detail here.

The longevity of log structures has always been a concern. It is partly dependent on tree species. Redwood, western red cedar, and Douglas fir may be expected to last the longest. Spruce, hemlock, white fir and pine are the least durable. The longevity will be improved when

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CHAPTER 4. CHANNEL BANK PROTECTION DESIGN

4.1 INTRODUCTION

This chapter is intended to provide design criteria and procedures for stream bank protection. As a steward of the watershed, SCVWD intends to use the "softest" or least environmentally impacting, and most environmentally beneficial, bank protection methods. Those methods will be highlighted in this chapter.

As has been described in other chapters of this manual, it is necessary to understand the process by which streams are formed and by which they adjust to changes to their environment, before tackling a bank protection project. Without an adequate understanding of the cause of bank erosion, repair solutions may well be inadequate to address the problem or adversely impacting the habitat. Hence, this chapter is organized in the same order as a design project, from field investigation, identifying the problem and cause, design considerations, selecting bank protection method, to design details.

Bank protection can be provided by many methods: from bank regrading, revegetation, all the way to channel hardscaping. The best design takes into consideration the bank failure mechanism, decides if any bank protection action should be taken, develops the type of bank protection best suited for the site, and optimizes the extent of the bank protection application. These actions take into account hydraulic, geomorphological, geotechnical, construction, maintenance, operation, and environmental issues. They also require careful field investigation and engineering analysis. These steps are described in the following.

4.2 FIELD INVESTIGATION

In order to develop an adequate background of the stream condition, the following information must be collected through field investigation and analysis:

Channel bankfull flow and dimensions, following the methodology of Chapters 9 and 1.

Channel sediment transport condition: determine if the project reach is aggrading, degrading, or stable, following methodology described in Chapter 10.

Local channel features: local "hard points" (channel invert stabilizers), storm drain outfalls, and bank irregularities (such as sharp bends, intruding points or construction debris left by property owners) in the vicinity of bank failure area.

Channel vegetation cover and potential for revegetation: determine if the local area can support native vegetation and if there is potential for vegetation establishment (access to sunlight, groundwater, and proper soil). This will require input from a botanist familiar with local conditions.

Local water table: determine irrigation needs.

Local soils: determine suitability of local soils for vegetation and resistance to erosion, as well as geotechnical bank stability.

Local habitat: determine if species of particular concern exist in potential work area. Investigate local laws regarding heritage trees, etc.

Work access: investigate potential access routes to the bank repair site. This may have a major impact on the types of bank repair that may be implemented.

The designer should spend adequate time to become familiar with the intangibles of the bank repair locality and, if possible, observe the creek in action during a flow event. Take photographs for record and design aid.

4.3 DESIGN CONSIDERATIONS

Following the field investigations, the project team should try to address these questions to arrive at an appropriate bank repair design:

What type of bank erosion is observed?

What is the cause of this bank erosion?

Should a bank stabilization project be conducted at the subject site?

What is the most appropriate bank protection method and boundary to be implemented?

These questions will be answered after the project team considers the hydraulic, geomorphic, geotechnical and environmental factors involved at the site. These factors will be discussed in the following.

As described in the beginning of this chapter, a general commitment to preserve the environment at SCVWD has prompted the adoption of environmentally friendly methods for watershed construction. A key element of this move is to use bioengineering bank-protection methods, i.e., methods that utilize natural materials such as logs and vegetation. Because these materials are biodegradable, there have been concerns of the “temporary” nature of these methods. It is, hence, important for a design engineer to understand the reasonable life span of a bioengineering structure and factor that into the overall design approach to meet project objectives.

Frissell and Nawa [1992] found that 60 percent of instream structures surveyed in southwest Oregon and southwest Washington were either damaged or destroyed by 2- to 10-year storm events. Hopelain [1998] reported that several log structures placed in 1987, or before, in northwestern California streams exhibited little sign of deterioration when surveyed in 1995. Olson [1990] evaluated condition of instream structures in Klamath River basin and estimated lifespan of 25 to 50 years for logs and 40 to 50 years for small boulder weirs. A review of these documents indicates that:

Most of the instream structures built in the 1980’s were not engineered.

These structures failed mostly due to flow and sediment conditions not considered in design

The life expectancy of logs and boulders is governed by external conditions, such as high flow event, uncontrolled sediment movement and poor construction quality, and not the materials themselves.

Based on these lessons learned, the design procedures provided below, and in Chapters 3 and 5 for grade control weirs and rock riprap structures, will include determination of design flow and proper sizing of log and boulder materials for installation to clearly define the design condition. For bank protection, the design intent is to protect the site with natural materials that can withstand the local hydraulic condition, plant the site to encourage vegetation establishment, and monitor the site condition to ensure proper performance. Given proper design, installation and monitoring, it is expected that a design life of more than 25 years should be achieved from these methods.

4.3.1 Design Flow

The design flow for bank protection measures depends largely on the type of channel geometry. For a stream having a bankfull channel with wide floodplains, it is adequate to extend bank protection only to the bankfull stage. This is because, as described in Section 9.6.1 and shown in Figure 9-12b(3), that the average bottom shear stress will decrease as flow exceeds the bankfull discharge. In addition, because of the small water depth on the floodplains, the shear stress on floodplains is typically much lower than the channel bottom.

On the other hand, in cases where the channel is entrenched due to previous down-cutting, and there is no floodplain available to the creek, the shear stress will continue to increase as flow goes above the bankfull discharge. It is then prudent to design bank protection to the 10 percent flow level. Flows higher than the 10 percent flood will occur, but at less frequent intervals. On top of this, it is also recognized that the shear stress on channel banks is on average about 20 percent less than that of the channel bottom, and the average shear stress on channel bottom will be used for bank protection design.

Based on these considerations, and for purposes of simplifying the design criteria, the design flow and protection level for bank protection project are determined as follows:

$$
\begin{aligned}
&\frac{W_{fp}}{W} \geq 1 \qquad Q_{design} = Q_{bf} \qquad H_{bp} = Y_{bf} \\
&\frac{W_{fp}}{W} < 1 \qquad Q_{design} = Q_{10\%} \qquad H_{bp} = Y_{10\%}
\end{aligned}
\tag{4-1}
$$

where Wfp is the floodplain width and W is the bankfull channel width as defined in Figure9-12b(1). Qdesign is the design discharge for bank protection, Qbf is the bankfull discharge, and Q10% is the 10% flow. Hbp is height of bank protection, Ybf is the bankfull stage, and Y10 percent is the water level at 10% flow.

Generally for bank protection in residential areas, it is rarely necessary to extend bank protection to higher than the 10 percent flow elevation. However, special situations do exist, e.g., bank protection in the vicinity of a highway, railroad crossing, or large buildings where human lives may be at risk upon bank failure, may warrant higher than 10-year bank protection. In that case, the project team should evaluate the situation, consult with stakeholders involved, and determine the design flow and protection level accordingly.

During construction (July, 2001) **During construction (July, 2001)**

Post-construction (November, 2001) **(March, 2002)**

Figure 4-3. Mild Slope Erosion and Repair on Alamitos Creek

4.5 BANK PROTECTION METHODS

The bank protection methods described in this design manual are separated into three groups, for toe, steep slope and mild slope erosions. The methods use mostly natural materials available in the watershed, logs, rocks and vegetation. Artificial materials such as concrete, concrete sacks, gunite and gabions are excluded from the manual to discourage their usage and because they will seldom be permitted by the regulatory agencies.

Only several of the most applicable methods for each erosion type are provided to simplify the design approach. These methods are also consistent with those provided in the Water Resources Protection Manual [SCVWD 2007]. The design manual also augments the Water Resources Protection Manual by providing an engineering procedure with relevant calculations to design the bank protection structures. Should there be a need to utilize other bank protection methods, an excellent reference is Biedenharn, et al. [1997].

In all cases, successful long term protection depends on establishment of the vegetation planted. Therefore, correct matching of the plant species with site conditions is important. A list of appropriate vegetation species for the Santa Clara Valley is provided in Section 4.6. The selection of plant species to be used at a site should still be made in coordination with a botanist knowledgeable of the native vegetation.

It is also important to realize that there are limitations to the application of each method. Some are limited by characteristics of the material, and others by the site conditions. Misusing a method may result in failure at a later time. These limitations are described in the design procedures that follow, and summarized in Table 4-1 for easy reference.

4.5.1 Toe Protection Methods

The basic bank protection methods for toe erosion include:

Rock toe protection;

Rootwad and rock toe protection; and

Log toe protection.

4.5.1.1 Rock Toe Protection

Rocks may be used to protect the toe of a bank. This is especially useful when the upper bank is stable because of vegetation, cohesive materials or low flow velocity, but the toe is eroded.

Design Procedure

1. Examine the site condition, historic survey data and maintenance record to determine if the site is at dynamic equilibrium, or if further erosion may be possible.

2. Determine design flow using Eq. (4-1).

3. Run HEC-RAS analysis to determine the local water depth, flow velocity and other hydraulic parameters.

4. Calculate potential local scour depth, if needed, at bridge pier, channel contraction or bend as the case may be, using methods described in Section 10.3.8 .

5. Determine design rock size and gradation using methods described in Section 5.2.1.

6. Measure in the field dimensions of the erosion cavity and elevations to prepare for repair design drawing.

7. Consult project biologist or botanist about planting needs to establish vegetation around site.

8. Prepare design drawing similar to Figure 4-4.

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CHAPTER 5. MISCELLANEOUS HYDRAULIC STRUCTURE DESIGNS AND CONSIDERATIONS

5.1 GENERAL

This chapter will cover design practices of several open-channel hydraulic structures used in flood protection and water utility operations but have not been discussed in previous chapters. These structures include rock riprap, detention basins, conduit outlets, and channel transitions.

Rock riprap structures are cited often in Chapters 4 and 3 to protect channel banks and bed and control grade. Rock riprap is possibly the most often used material for channel protection. Detention basins are used in flood protection projects to detain or retain floodwater in upstream locations to reduce peak flow so that the downstream reach may be more effectively protected. Detention basins are also key elements to development projects to meet requirements of the hydromodification management plan mandated by all Bay Area cities and counties.

Conduit outlets are used at the ends of tunnels and pipelines to dissipate excess energy and transition the flow regime smoothly to open channel in the downstream. With urbanization, storm drain outlets have also become a significant cause of local erosion in streams and they need to be designed properly to prevent undue damage to the creeks. Channel transitions may be problematic for water supply and flood protection projects and should be designed carefully.

The procedures to analyze and design these hydraulic structures are described in the following. At the end special considerations for boundary conditions, risk-based analysis and freeboard requirements are also provided.

5.2 RIPRAP PROTECTION DESIGN

The riprap design is provided in three sections. First, Section 5.2.1 discusses methods of determining the rock size, thickness, and gradation. Section 5.2.2 then discusses design of the filter layer underneath the rock riprap. Finally, Section 5.2.3 discusses design of the grouted rock riprap.

5.2.1 Rock Sizing

There are numerous riprap design methods in the literature. After a literature search, several prominent design methods were identified which include USACE [1994], Caltrans [2000], Isbash [1936], Shields [1936], and ASCE [1975]. These design methods are grouped into several applications to facilitate analysis and selection of design method. The applications consist of channel bank protection, rock weir design, scour pool sizing and stilling basin design. The basis of data and limitations of each method are provided to further clarify conditions of application. Riprap gradation and filtering material designs are also described to complete the design procedure.

5.2.1.1 Channel Bed and Bank Protection

The hydraulic setting of this application is basically flow parallel to the riprap surface. For bed protection, the riprap surface is horizontal, and the shear force is the only external force acting

Table 5-1
Gradations for Riprap Placement in Low Turbulence Zones From USACE [1994]

Layer Thickness, ft	$W_{100(max)}$, lb	$W_{100(min)}$, lb	$D_{100(max)}$, ft	$D_{100(min)}$, ft	$W_{50(max)}$, lb	$W_{50(min)}$, lb	$D_{50(max)}$, ft	$D_{50(min)}$, ft	$W_{15(max)}$, lb	$W_{15(min)}$, lb	$D_{15(max)}$, ft	$D_{15(min)}$, ft	$D_{30(min)}$, ft	$D_{90(min)}$, ft
0.75	36	15	0.75	0.56	11	7	0.50	0.43	5	2	0.39	0.29	0.37	0.53
1.00	86	35	1.00	0.74	26	17	0.67	0.58	13	5	0.53	0.39	0.48	0.70
1.25	169	67	1.25	0.92	50	34	0.83	0.73	25	11	0.66	0.50	0.61	0.88
1.50	292	117	1.50	1.11	86	58	1.00	0.88	43	18	0.79	0.59	0.73	1.06
1.75	463	185	1.75	1.29	137	93	1.17	1.02	69	29	0.93	0.70	0.85	1.23
2.00	691	276	2.00	1.47	205	138	1.33	1.17	102	43	1.06	0.79	0.97	1.40
2.25	984	394	2.25	1.66	292	197	1.50	1.32	146	62	1.19	0.90	1.10	1.59
2.50	1350	540	2.50	1.84	400	270	1.67	1.46	200	84	1.32	0.99	1.22	1.77
2.75	1797	719	2.75	2.03	532	359	1.83	1.61	266	112	1.45	1.09	1.34	1.96
3.00	2331	933	3.00	2.21	691	467	2.00	1.75	346	146	1.59	1.19	1.46	2.11
3.50	3704	1482	3.50	2.58	1098	741	2.33	2.05	549	232	1.85	1.39	1.70	2.47
4.00	5529	2212	4.00	2.94	1638	1106	2.66	2.34	819	346	2.11	1.59	1.95	2.82
4.50	7873	3149	4.50	3.31	2335	1575	3.00	2.63	1168	492	2.38	1.79	2.19	3.17

Caltrans [2000]

The California Department of Transportation developed the California Bank and Shore Rock Slope Protection design method to protect riverine highway embankments. The rock sizing equation was originally developed in 1960. Through the years, continual applications and field experiences cumulated. Caltrans engineers conducted systematic investigations into past site applications and maintenance reports to improve the design method to include multiple layers and resulted in the current method described in Caltrans [2000].

Rock Sizing

$$W = \frac{0.00002 V^6 G_s}{(G_s - 1)^3 \sin^3(\varphi - \theta)} \tag{5-5}$$

where

W = theoretical minimum rock weight, lb

V = velocity to which bank is exposed, ft/s

= 0.67 times the channel average velocity for parallel flow

= 1.33 times the channel average velocity for impinging flow around bends

G_s = specific gravity of rock

φ = angle of repose for randomly placed rubble, use 40 for rounded river rocks

θ = angle of outside slope with horizontal, degrees

Eq. (5-5) calculates the rock weight that will be sufficient to resist the flow and remain stable. This weight is then entered into Table 5-2 from the left to determine the gradation for this outside layer.

Gradation

Table 5-2
Outside Layer Design for Caltrans [2000]

STANDARD Rock SIZE or Rock MASS or Rock WEIGHT		GRADING OF ROCK SLOPE PROTECTION **PERCENTAGE LARGER THAN**											
		RSP-Classes [A]											
		Method A Placement					Method B Placement						
		RSP-Classes other than Backing									Backing No.		
US unit		8 ton	4 ton	2 ton	1 ton	1/2 ton	1 ton	1/2 ton	1/4 ton	Light	1 [B]	2	3
	SI unit	8 T	4 T	2 T	1 T	1/2 T	1 T	1/2 T	1/4 T	Light	1 [B]	2	3
16 ton	14.5 tonne	0-5											
8 ton	7.25 tonne	50-100	0-5										
4 ton	3.6 tonne	95-100	50-100	0-5									
2 ton	1.8 tonne		95-100	50-100	0-5		0-5						
1 ton	900 kg			95-100	50-100	0-5	50-100	0-5					
1/2 ton	450 kg				95-100	50-100	-----	50-100	0-5				
1/4 ton	220 kg					95-100	95-100	-----	50-100	0-5			
200 lb	90 kg							95-100	-----	50-100	0-5		
75 lb	34 kg								95-100	-----	50-100	0-5	
25 lb	11 kg									95-100	90-100	25-75	0-5
5 lb	2.2 kg											90-100	25-75
1 lb	0.4 kg												90-100

[A] US customary names (units) of RSP-Classes listed above SI names, example US is "2 ton" metric is "2 T".
[B] "Facing" has same gradation as "Backing No. 1". To conserve space "Facing" is not shown .

For example, if Eq. (5-5) calculates a rock weight of 850 lbs, the nearest rock size that is greater in the left column is ½ ton. In the row of ½ ton, go to the cell where 50-100 percent is larger and go up to find that the rock class is ½ T, and the gradation is 0-5 percent is greater than 1 ton, 50 – 100 percent is greater than ½ ton, and 95-100 percent is greater than ¼ ton. This is the outside layer design. Table 5-3 then defines the requirements for inner layer, if applicable, and backing and fabric layers for filtration. A standard design includes the outer, inner, backing and fabric layers.

In Table 5-2, the method A of placement is to place stones individually so that there is 3-point contact on adjacent rocks and no wobbling. It is specified for RSP (rock slope protection) class ½ ton or higher. Method B is to place rocks by dumping from near their planned location and then use machinery to work stones to their final position.

In Table 5-3 the fabric layer may be replaced by 230-mm of backing layer No. 3. The fabric types A and B are defined in standard Caltrans Specifications Section 72, with type A having less mass per unit area and less toughness than type B, but both having a minimum permittivity of 0.5 per second.

Table 5-3
Multi-Layer Design for Caltrans [2000]

OUTSIDE LAYER RSP-CLASS *	INNER LAYERS RSP-CLASS *	BACKING CLASS No. *	RSP-FABRIC TYPE **
8 T (8 ton)	2 T over 1/2 T	1	B
8 T (8 ton)	1 T over 1/4 T	1 or 2	B
4 T (4 ton)	1/2 T	1	B
4 T (4 ton)	1 T over 1/4 T	1 or 2	B
2 T (2 ton)	1/2 T	1	B
2 T (2 ton)	1/4 T	1 or 2	B
1 T (1 ton)	LIGHT	NONE	B
1 T (1 ton)	1/4 T	1 or 2	B
1/2 T (1/2 ton)	NONE	1	B
1/4 T (1/4 ton)	NONE	1 or 2	A
LIGHT (LIGHT)	NONE	NONE	A
Backing No. 1*** (Backing No. 1)	NONE	NONE	A

Layer Thickness

This multiple layered design is aimed to increase interlocking and filtration between layers, but will often result in a riprap layer of 5 – 10 ft in thickness, as specified in Table 5-4 for minimum layer thickness. This may be suitable for large rivers with wide open channels. For some urbanized streams in Santa Clara Valley, where channel width is small and restrictions in real estate often makes cut-and-fill infeasible, this method may not be conveniently applied.

Table 5-4
Minimum Layer Thickness for Caltrans [2000] Riprap Design

RSP-Class Layer	Method of Placement	Minimum Thickness
8 T (8 ton)	A	2.60 meters (8.5 feet)
4 T (4 ton)	A	2.07 meters (6.8 feet)
2 T (2 ton)	A	1.65 meters (5.4 feet)
1 T (1 ton)	A	1.31 meters (4.3 feet)
1/2 T (1/2 ton)	A	1.04 meters (3.4 feet)
1 T (1 ton)	B	1.65 meters (5.4 feet)
1/2 T (1/2 ton)	B	1.31 meters (4.3 feet)
1/4 T (1/4 ton)	B	1.00 meters (3.3 feet)
Light	B	760 millimeters (2.5 feet)
Facing	B	550 millimeters (1.8 feet)
Backing No. 1	B	550 millimeters (1.8 feet)
Backing No. 2	B	380 millimeters (1.25 feet)
Backing No. 3	B	230 millimeters (0.75 feet)

ASCE [1975]

The ASCE method uses the Isbash equation with a modification for channel bank slope changes.

$$W = \frac{0.000041 G_s V^6}{(G_s - 1)^3 \cos^3 \theta} \tag{5-6}$$

The same definition for variables as Caltrans [2000] is used here. The same Eq. (5-4) is used to convert rock weight to size (diameter). The rock sizing Eq. (5-6) is essentially the same as the Caltrans Eq. (5-5).

Design Procedure for Bank and Bed Riprap Protection

Williams [2000] evaluated 7 design methods including USACE, Caltrans and ASCE and found all three methods described above acceptable, with the USACE method in particular showing more sensitivity to the inherent instability of steep bank slopes. The recommended design procedure is as follows:

1. Use the USACE method to calculate the D_{50} rock size, determine riprap thickness, and specify gradation

2. Use the Caltrans or ASCE method to compute rock size, and compare the result with that of USACE. This exercise serves to verify the selection of safety factor and other coefficients used in USACE.

3. Determine filter design using procedures of Sections 5.2.2.

5.2.1.2 Rock Weir

In addition to the shear force experienced by the rocks in bank or bed riprap, the rock weir will experience hydrodynamic forces of impingement and turbulence resulting from the bed drop downstream. To account for this flow condition, the Corps of Engineers' equation for steep slope riprap design [USACE 1994] may be used. Thomas et al. [2000] also found this equation applicable in sizing weir rocks in a step-pool arrangement. The step-pools are series of drops and pools which are often observed in mountainous streams of slopes from 2 to 10%. The drops are essentially rock weirs. The range of slopes is similar to that for which the steep-slope riprap design equation, i.e., Equation (5-7) below, was developed. That is perhaps one of the reasons why the steep-slope riprap equation has been found suitable for the rock weir design.

Rock Sizing

$$D_{30} \qquad \frac{1.95 S^{0.555} q^{2/3}}{g^{1/3}} \qquad (5\text{-}7)$$

Where

q = unit discharge of design flow Q_{design} determined from Eq. (3-1)

= Q_{design}/W, where *W* is bankfull width

S = slope of channel bed

Use Eq. (5-2) to convert D_{30} to D_{50}. Determine D_{100} from Table 5-1.

Gradation

Specify a uniform gradation ($D_{85}/D_{15} < 1.4$) for the rocks.

Thomas et al. [2000] installed four step-pools in 1989 in the San Miguel River where the slope was approximately 0.6%, about an order of magnitude smaller than the natural streams observed with step-pools. The structures have withstood several large flows since then without significant damage.

5.2.1.3 Scour Pool

The scour pool is a depressed area downstream from a small drop structure where the potential energy from the drop may be dissipated through turbulence. A scour pool may be formed downstream of a drop structure, a concrete channel, or a pipeline outlet. In natural streams this bed drop and scour pool combination is commonly observed in mountainous streams made of gravels and cobbles and of steep longitudinal slopes. It is also often seen in the upper watersheds of the Santa Clara Valley. In a natural setting, it is called a step-pool structure. Thomas et al. [2000] collected step-pool data from 8 streams in Colorado, and developed empirical equations for step pool geometry. These geometric dimensions are adopted here to form the scour pool downstream of a rock weir. The stones that line the scour pool are sized using the Isbash [1936] equation.

Isbash [1936]

The Isbash method was developed for the construction of dams. Isbash [1936] published coefficients for the stability of rounded stones dropped in running water. These coefficients were later compared by the Corps of Engineers to Bonneville Hydraulic Laboratory and Waterways Experimentation Station laboratory data to show satisfactory match.

$$D = \frac{V^2}{2g\,C^2\,(G_s - 1)} \tag{5-8}$$

where

D = stone size, ft

V = average channel velocity, ft/s

G_s = specific gravity of rock ($_s$/ $_w$)

g = gravitational acceleration, ft/s^2

C = Isbash coefficient

= 0.86 for high turbulence zones

= 1.20 for low turbulence zones

The Isbash coefficients were determined from tests with no boundary layer development and the average velocity was representative of the velocity against rock. When the rock movement resulted from sliding, a coefficient of 0.86 was obtained. When the movement was effected by rolling or overturning, a coefficient of 1.20 was obtained. Later the Corps of Engineers associated the coefficient 0.86 with high turbulence area, such as a stilling basin, and the coefficient 1.20 with low turbulence such as river closure.

The flow condition for a low-drop scour pool arrangement is similar to that of the low turbulence condition, especially under design flow condition when the weir flow is drowned, and the Isbash coefficient of 1.20 is appropriate to size the rocks in the pool.

Thickness

$$T = 1.5\ D_{100(min)} \qquad (5\text{-}9)$$

Gradation

Use the same gradation design as specified in Table 5-1. Design the filter layer following procedures in Section 5.2.2.

Appendix E
Fish Passage Data Provided by SCVWD

Low Flow Channel Length vs Maximum Average Water Velocity for Adult Steelhead	
Culvert Length (ft)	Adult Steelhead (fps)
<60	6
60-100	5
100-200	4
200-300	3
>300	2